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Geotechnical Consulting

GEOTECHNICAL INVESTIGATION REPORT
for the
Proposed Bohemia Residential Development
Auburn, California

Submitted to

Tim Lewis Communities
5750 Sunrise Blvd, Suite 225
Citrus Heights, CA 95610

by

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December 02, 2004

Mr. Jim Migliore
Tim Lewis Communities
5750 Sunrise Blvd, Suite 225
Citrus Heights, CA 95610

Subject: **GEOTECHNICAL INVESTIGATION REPORT**
for the Proposed Bohemia Residential Development
Auburn, California
EGC Project No. P201

Dear Jim:

Enclosed is our Geotechnical Investigation Report for the proposed Bohemia Residential Development located in Auburn, California. Our work was performed in general accordance with our revised proposal dated October 2, 2004. The results of our investigation are included in this report.

Our geotechnical investigation was performed to evaluate the subsurface conditions at the subject site and provide recommendations for design and construction of the proposed improvements. In general, the site soil conditions were found to be suitable for conventional residential building and pavement construction. However, the site is underlain by very dense by metavolcanic rocks (commonly referred to as greenstone) to the east and serpentine to the west, where very difficult excavation is expected and special engineered fill construction techniques will be required.

Geotechnical studies using a limited number of exploratory test pits relies on an assumption of uniformity of soil between test pits; often during construction we find this not to be the case. Therefore, in presenting this report we do so with the understanding that we will be allowed to continue on this project by providing observation and testing services during construction to verify the materials encountered in our investigation.

Sincerely,

ESPAÑA GEOTECHNICAL CONSULTING

Michael Wilson
Staff Geologist

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INTRODUCTION

Location and Description of Project

The project is composed of an approximately 18-acre site located off Canal Road approximately 2,000 feet north of Luther Road, near the City of Auburn, County of Placer, California.

Although the property has not been purchased, the project is anticipated to include about 110 single-family homes is anticipated that the building construction will be lightly to moderately loaded, wood-framed structures. Roadways and appurtenant underground utilities will also be included. Due to the sloping nature of the site it is anticipated that substantial cuts and fills may be necessary as well as relocating and/or enclosing the existing Fiddler Green Canal. Some retaining walls may also be required, especially near the northern edge of the property adjacent to the existing residential development.

Purpose and Scope

This Geotechnical Investigation Report (GIR), prepared by España Geotechnical Consulting (EGC), is intended for use by the architects and engineers involved in the Bohemia Residential Development project. The purpose of our work was to perform a geotechnical investigation which included trenching, logging, sampling, and laboratory testing to generate recommendations for grading, foundation design, concrete flatwork placement and pavement construction. Based on our understanding of the proposed improvements and site conditions, we have performed the following scope of work:

1. Excavated 16 test pits at selected locations throughout the site using a rubber-tired backhoe. Bulk soil samples were collected from the test pits for laboratory analysis.

2. Performed laboratory testing of selected soil samples to classify the soil and determine general strength parameters. Tests have included grain-size analysis, R-value testing, and corrosion potential.

3. Performed analytical testing on acquired serpentine rock to determine the presence of naturally occurring asbestos (NOA).

4. Prepared a report summarizing our findings, conclusions; and recommendations for site development. Recommendations include:

    a. Site clearing and preparation
    b. Site grading
    c. Utility installation
    d. General soil corrosivity
    e. Foundation design
    f. Flatwork and slab-on-grade design
    g. Retaining wall design parameters
    h. Pavement Recommendations
    i. Recommendations for NOA

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices in the greater Sacramento region. This warranty is in lieu of all other warranties, either expressed or implied.

SITE CONDITIONS

Geology

Published geologic literature indicates that the study area is underlain by Paleozoic to Mesozoic metavolcanic rocks (common by referred as greenstone) and ultramatic rocks (serpentine) (Loyd, 1995; Wagner, D.L., et al, 1981; Livingston, 1974). The surface of the greenstone typically weathers to a reddish clay and clayey and silty sand while the soils covering the serpentine weather to a light to moderately dark green silty and sandy clay. Based on our field investigation at the site the soil overlying the greenstone is typically thicker than those overlying the serpentine.

Surface

The property slopes generally downhill to the west-southwest towards State Route 49 and consists of a series of relatively flat terraces separated by the Fiddler Green Canal and the Wise Canal. It appears that past grading has occurred on this site to create the terraces for the former lumber mill operation. The eastern half of the proposed site (east of the Fiddler Green Canal) is generally grass-covered with occasional trees. Weathered asphalt pavement and the remnants of the slabs and foundations of the former lumber mill structures generally cover the western half of the proposed property, between the Fiddler Green Canal and the Wise Canal. Several 55-gallon drums and scrap metal were observed in the south-central portion of the site near where the Wise Canal intersects the west edge of the PG&E property.

Subsurface Soil Conditions

Test Pits

Excavation of 16 test pits using a rubber-tired backhoe were performed. The soil varies from approximately ½ to 5 feet in thickness, and is composed of red to tan sandy clay/clayey sand with residual rock fragments. The materials are typically weak (moderately stiff or medium dense) near the surface, and grade to relatively competent material at between 2 to 5 feet below existing grade. The surface soil is underlain by greenstone that varies from completely weathered and sheared (residual soil with remnant rock fabric) to fresh, unweathered, hard greenstone.

The western portion of the site was typically covered by weathered asphalt pavement, gravel or concrete. Below the asphalt or gravel pavement and underlying aggregate baserock the soils (where encountered in Test Pit T-18) consisted of red, rocky silty and clayey sand which graded to a tan and green moderately weathered to relatively unweathered serpentine.

The excavatability of the upper soils ranged from relatively easy to moderately difficult using a conventional rubber-tired backhoe. Excavation refusal was encountered on the very dense relatively unweathered greenstone and serpentine at depths which ranged from as shallow as 1 foot bgs in Test Pit

T-10 to as deep as 5 feet in Test Pits T-5 and T-11. Outcrops of greenstone were observed at the ground surface in the southeast portion of the subject site and exposed serpentine was encountered along a scarp in the west-central portion of the project area.

Test Pits T-12 and T-13 were excavated in the south-central portion of the site in an area that had the surface expression of fill These test pits confirmed the presence of fill to depths of 3 to 4 feet. The fill at both locations generally consisted of relatively loose clayey and silty sand with occasional rock fragments and trace roots, wood and chunks of concrete. Test pits T-16 and T-17 were eliminated due to the inability to excavate through the concrete pavement. Materials encountered during our subsurface investigation in the eastern half are consistent with the published mapping, site observations and subsurface excavations performed by others at the site. In general, the site is underlain by soil and rock.

The soil profiles described above are generalized; therefore, if the soil conditions at a specific location are desired, consult the logs of test pits, Appendix I, Figures I-1 through I-16. The Test Pit Legend is provided as Figure I-17 and the Rock Classification System is included as Figure I-18. On the test pit logs, the soil type, color, moisture, and Unified Soil Classification System (USCS) symbol are indicated. Boring and test pit logs by others are presented in Appendix IV.

Materials encountered during our subsurface investigation in the eastern half are consistent with the published mapping, site observations, and subsurface excavations performed by others at the site.

The locations of test pits were determined by locating them in the field using existing landmarks. Actual locations of the test pits may be surveyed in the field at a later date; however, for this report, the elevations of the test pits were not shown. Accuracy of the test pit locations can only be implied to the degree that these methods warrant.

The test pits were loosely backfilled and wheel-rolled. We assume this to be the case for the test pits by others as well. Wherever the test pits (ours and by others) will not be removed during grading and are within the planned development limits, the test pits should be over-excavated and backfilled with compacted fill. The fill should be compacted to the standards described in this report.

Seismic Refraction Survey

A seismic refraction survey was perform at six select locations throughout the subject site to help determine the potential excavation characteristics of the underlying bedrock. The seismic refraction survey was performed by EGC personnel on November 18, 2004 using a Smartseis LS12 seismograph. Each seismic refraction survey was performed along a 120 foot traverse using 12 geophones (10 foot spacing) and a striker plate and 10 pound sledge hammer as the seismic source. The location of each traverse are shown on Figure 2, Test Location Plan and the results of our seismic refraction survey are presented in Appendix III.

The results of our seismic refraction survey generally indicated primary wave velocities thickness of soil over rock material. The transverses indicate soil thickness of up to 4 feet and primary wave velocities of up to 2,500 feet per second (fps). Similarly the rock below the soil had primary wave velocities of 2,500 to 20,000 fps. These values are consistent with the seismic refraction data presented by others for the northeast corner of the site. The soil and rock parameters described above are generalized. If conditions at a specific location are desired, consult Appendix III, Table III-1.

Groundwater

Groundwater was not encountered in our shallow test pits during our investigation in November 2004. Nor was ground water encountered in previous borings by others performed on this site, which were advanced to depths up to 20 feet during July 1992 (See Appendix IV). Due to the dense to very dense nature of the underlying bedrock it is likely however that groundwater seepage may be encountered at the rock-soil interface, especially during the wetter months.

Groundwater observations were made in the test pits at the times and under the conditions stated. It should be noted that fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, irrigation, and other factors.

LABORATORY TEST RESULTS

Geotechnical Testing

Selected samples obtained during our field investigation were tested to determine a variety of physical properties. The test types and procedures used are presented in Appendix II. The test results are presented in Appendix I on the Logs, and in Appendix II in Tables II-1 and II-2 and Figures II-1 and II-2. A summary of the laboratory test results is presented below.

Grain size distribution tests and Atterberg Limits tests were performed on a typical samples of soil retrieved from the test pits to aid in visual classification.

Two samples of soil anticipated to be in contact with the new structures and pipelines were tested for sulfate and chloride content and pH and resistivity. The pH for the samples ranged from 6.4 to 6.5, and minimum resistivities were 2,456 to 4,158 ohms-cm. Sulfate and chloride test results were less than 25 ppm/dry weight.

An R-value test, run on composite bulk samples of the near-surface soil, resulted in an R-value of 42.

For the purpose of determining the presence of naturally occurring asbestos, mineralogic analysis was performed on selected rock samples. Asbestos-type minerals looked for include actinolite, tremolite and chrysotile. The testing procedures used are also presented in Appendix II. The test results are presented in Table II-2.

Naturally Occurring Asbestos (NOA) Testing

Naturally occurring asbestos is typically present within local zones of ultramafic rock, typically serpentinite. The ultramafic rocks are thought to be associated with fault complexes located in the western foothills.

Ultramafic (serpentine) rocks occur in the general area of the site, and were observed in the western portion of the site during our field investigation. These serpentine (or serpentinite) rocks are commonly associated with a group of NOA minerals, such as chrysotile, actinolite, and tremolite. With the exception of a trace amount (less than 1 percent) of chrysotile in one collected sample, mineralogic analysis (see Appendix II) did not reveal the presence of NOA in the rock samples collected from the site.

CONCLUSIONS AND RECOMMENDATIONS

Expansive Soil

Visual classification of the site soils and Atterberg Limits tests indicate that the soil expansion potential is considered low; therefore, no special recommendations have been provided for expansive soil conditions. However, the project geotechnical engineer or their representative should observe the grading and foundation excavations so that potentially expansive soil conditions could be identified should they be exposed.

Soil Corrosion Potential

The results of our corrosivity testing indicate that the majority of the site soils have a slightly acidic pH, and have moderate resistivity. Sulfates and chlorides were low. This could indicate a mildly corrosive environment to ferrous metals, but we recommend a corrosion engineer be retained to review the results of our corrosivity tests and to provide recommendations for protection of the specific types of buried facilities planned for the site.

Naturally Occurring Asbestos

Exposed rock at the site are moderately to intensely weathered, metavolcanic and ultramafic rock. Local occurrences of NOA are typically associated with faulting and ultramafic rocks. Ultramafic rock was identified during our site reconnaissance, generally in the western portion of the site. Rock samples acquired from our subsurface field investigation, were submitted to a certified laboratory for asbestos identification and analysis.

With the exception of a trace amount (less than 1 percent) of chrysotile, the mineralogical testing of the rock samples did not detect the presence of asbestos minerals. Special grading requirements for soil/rock, as required by Placer County Environmental Health Department (PCEHD) where NOA minerals are present, will likely not be needed during grading and other site improvements. Due to the limited nature of our exploration, however, the possibility of encountering localized areas of asbestos containing rock during grading cannot be ruled out. Our geologist should be on-site during mass grading to continue the presence of asbestos containing rock.

Levels of Shaking and Seismicity

A complete seismic analysis was beyond the scope of this investigation, however our limited research indicated that the site is located within the Foothills Fault System boundaries. The site is not mapped as being within an Alquist-Priolo Earthquake Fault Zone.

Plan Review and Construction Observation

Preliminary earthwork and foundation recommendations for use in design and construction of the project are presented below. We recommend that our firm review the final design and specifications to check that the earthwork and foundation recommendations presented in this report have been properly interpreted and implemented in the design and specifications. We can assume no responsibility for misinterpretation of our recommendations if we do not review the plans and specifications.

The analysis, designs, opinions, and recommendations submitted in this report are based in part upon the data obtained from the test pits, seismic refraction survey, and upon the conditions existing when services were performed. Subsurface conditions varying from those analyzed or characterized in this report are possible and may become evident during construction. If variations are encountered during construction, it may be advisable to re-evaluate certain analyses or assumptions. Changes in the condition of the site can occur over time as a result of either natural processes or human activity and should be anticipated.

We recommend that our firm be retained to provide geotechnical and geologic services during site grading and foundation installation, to observe compliance with the design concepts, specifications, and recommendations presented in this report. Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered. If we are not retained to provide geotechnical services during site grading and foundation installation, we cannot be held liable for geotechnical-related problems.

Site Grading

General

Due to the lack of a deep soil horizon on the site, it may be necessary to import soils for landscape surfacing. Soils may be made available to the project by stripping off the surface soil and stockpiling these materials for later use.

The presence of shallow groundwater is not anticipated; however, if grading commences in the early spring or after a period of heavy rainfall, the surface soil may become saturated due to underlying, relatively low permeability soil/rock trapping water near the surface. This may create loading, hauling, and fill placement difficulties. Often, a period of at least a month after the last heavy rain of the season is necessary to allow the surface soil to dry sufficiently so that heavy grading equipment can operate effectively. If existing fill is encountered during construction, it should be identified and appropriately contained. Our representative should be present during grading to confirm the presence of groundwater and to provide supplemental recommendations.

The south-central portion of the site has an area of unconsolidated fill which contains appreciable amounts of rubble. Prior to grading in this area, the existing fill material should be removed. This area may then be filled with engineered fill material in accordance with the recommendations below.

Rippability

The results of the seismic refraction traverses indicate that the rock has primary wave velocities of about 2,500 to 20,000 feet per seconds (fps). According to the Caterpillar Performance handbook, edition 26, sedimentary rock greenstone with velocities of 8,000 to 10,000 fps are considered to be marginally rippable using a Cat D9R with a multi or single shank No. 9 ripper. Since primary wave velocities of greater than 10,000 fps were encountered localized zones of very resistant rock, however, should be anticipated. Very resistant rock with high primary wave velocities will require pneumatic hammers, hoe rams, or blasting. A contingency should be placed in the construction budget for these resistant conditions joint spacing in the rock varies from a few inches to as much as a few feet. Also, large boulders will probably be uncovered during the excavation process. The removed rock may be used as fill provided it is crushed to meet the size requirements for fill presented below in *Fill Material*.

Naturally Occurring Asbestos and Special Grading Requirements

Although only trace amounts of NOA were identified in our laboratory testing program, the probability of greater amounts NOA being encountered during grading operations cannot be excluded based on our limited number of subsurface investigation points and rock NOA tests. The contractor should be familiar with Placer County's special grading requirements and be prepare to implement them if greater quanties of NOA are encountered during construction. A geologist trained in the recognition of NOA should be intermittently present during grading operations, to observe site conditions and to specify if special grading conditions should be implemented due to the presence of NOA. If NOA is present, dust suppression, burial, and covering will be required by PCEHD. To further confirm the lack of NOA at the site, additional sampling and testing should be performed prior to construction.

Clearing and Stripping

Grading preparation should include removal of all vegetation, trees, fill material, existing foundations, test pit backfill, surface debris, and any saturated (yielding or pumping) soils prior to beginning site grading. Following general clearing operations, grasses should be stripped from the surface of the site. Stripping should extend to a depth of 2 to 3 inches below the surface. Strippings are not to be used within structural or pavement fills. Strippings could be stockpiled and used as topsoil in nonstructural areas such as landscape or ballfield areas (if acceptable to the landscape architect). Strippings used as fill in landscape areas should be placed in the uppermost portion of the fill and should not exceed 2 feet in total thickness. Structure locations should be verified prior to placement of strippings in close proximity to any structures.

Any designated trees and their associated roots should be removed such that there are no roots greater than 1 inch in diameter remaining. Excavations created from tree/root removal should be backfilled in accordance with the compaction recommendations below. Our representative should be present during tree/root removal to confirm the roots have been properly removed and to observe and test backfill placement.

Several concrete building slabs and foundations and weathered asphalt/concrete roadways still exist on the site. The foundations should be removed under our observation. Based on visual examination, steel reinforcement (rebar) was seen exposed through the concrete at various locations. It is likely that much of the existing concrete contains steel reinforcement and therefore would be difficult to process for reuse as controlled fill. Accordingly, it is expected that the concrete foundations and slabs that remain will require removal from the site. It is possible however, that the weathered asphalt pavement may be removed, crushed and processed for use as controlled fill.

It is assumed that the existing Fiddler Green Canal will need to be relocated or replaced with a closed culvert system. The current canal is a concrete-lined channel that currently conveys water. Prior to site grading the canal structure will require removal. If the soil below the concrete is saturated, it should be removed prior to any pipe placement or backfilling. It is not expected that the concrete will be able to be readily processed for reuse and will likely require removal from the site.

The test pits were loosely backfilled and wheel-rolled. We assume this to be the case for the test pits by others as well. Wherever the test pits (ours and by others) will not be removed during grading and are within the planned development limits, the test pits should be over-excavated and backfilled with compacted fill. The fill should be compacted to the standards described in this report.

Fill Material

Native soil and on-site crushed rock, which do not contain concentrations of organic matter and debris should be suitable for use as fill material, provided the rock are broken down to the sizes discussed below. Due to the appreciable amount of over-sized material (greater than 6 inches in diameter), the importation of off-site soil should be anticipated or at least included as an add item in the contractor's bid. Imported soil required for use as engineered fill should have less than 40 percent passing the #200 sieve (by weight), a Plasticity Index of no greater than 15, a particle size not greater than 6 inches and no more than 15% greater than 3 inches, and an R-value greater than 30. Potential borrow sources should be tested and approved prior to importing to the site.

It can be expected that up to 50 percent or more of the excavated materials may be unsuitable for reuse as fill due to the size, shape and nature of the materials (rock greater than 6 inches). Depending on the final grading plans, off-site disposal of the unsuitable materials should be anticipated. As an alternative to off-site disposal, the contractor may elect to use a rock crusher or similar suitable equipment to process the unsuitable material into a form which meets the criteria for fill in this report. The unsuitable materials (rock greater than 6 inches) are generally of an igneous and metamorphic origin and are considered very hard to extremely hard in nature. Large, heavy-duty rock crushing equipment will be required to crush this material if it is to be used on-site as approved fill material.

For engineered fills constructed with excavated rock and rock-like material, the rock and rock-like material should be processed to a maximum size of 6 inches. Rock and rock-like material fragments larger than 6 inches in any dimension should be broken down in size or used in nonstructural fills or specific locations approved by the project geotechnical engineer and resident engineer. In general, rock and rock-like material exceeding 6 inches in any dimension should be placed in approved locations at depths greater than 2 feet below finish grade. Large rock and rock-like material, over 12 inches across and up to a maximum of 36 inches in any dimension, may only be placed in fill at approved locations approximately 5 feet or more below finish grade. Again, based on the anticipated site grades, it is not expected that the project will have a significant amount of deep fills to allow the use of over-size material in engineered fills.

Cut and Fill Pads

The proposed grading should be designed so that no more than 5 feet of differential fill thickness exists below foundations. If any portion of a foundation is bearing on cut and other portions of the foundation are bearing on compacted fill, we recommend that the portion of the foundation bearing on cut be overexcavated at least 3 feet so that the entire foundation is bearing on compacted fill. Deeper over excavation may be necessary in order to satisfy the differential fill thickness recommendations. No foundation slab should be allowed to be supported on both fill and cut. Building pads should be entirely composed of either properly compacted engineered fill, competent native soils, or competent native rock. In no case should a building pad be composed of more than one of these materials. If any portion of an exposed building pad subgrade is composed of dissimilar materials, then the pad should be overexcavated as described above.

Site Preparation

Following general site clearing and stripping, excavations or depressions extending below planned finished subgrade levels should be cleaned to a firm soil base and should then be backfilled with engineered fill placed in accordance with the following recommendations.

Areas designated to receive engineered fill for support of structures and pavements, which have soil materials (not the hard rock) exposed on the surface, should be scarified to a depth of 12 inches or to refusal on rock (whichever depth is less), properly moisture conditioned to near the optimum moisture content, and compacted to a minimum of 90 percent relative compaction based on ASTM Test Method D1557, latest edition.

Final subgrades that are composed of hard rock may be left undisturbed, but should be cleared of all loose soils and rock fragments exceeding 6 inches in size. Any voids created by the removal of loose soil or rock fragments should be filled and compacted with approved on-site and/or imported soils.

It is assumed that the existing Fiddler Green Canal will require relocation or burial as a closed culvert system. Following diversion of the water and removal of the concrete channel lining, all soft, saturated or otherwise unsuitable materials within the former channel should be removed until firm stable conditions are encountered. Fills used to backfill pipe placed within the former channel, or to fill sections of abandoned channel, should be filled and compacted with approved on-site and/or imported soils. Fills should be compacted to a minimum of 90 percent relative compaction based on ASTM Test Method D1557, latest edition.

Depending on the final grading plans, removal of the underlying bedrock material may require removal to achieve plan subgrades. Due to the very dense nature of the bedrock materials it is possible that blasting may be required for removal. It is recommended that the contractor include a contingency for blasting for rock removal.

Compaction

General

Rock and rock-like material must be placed so that soil may be compacted around the rock; rock-to-rock contact should be avoided and large rock and rock-like material must be separated so that compaction equipment can compact the soil around the rock. Large rock should not be placed where they could interfere with future utilities or below-grade structures.

All structural fills should be placed in horizontal lifts not exceeding 8 inches in compacted thickness. Each lift should be compacted to a minimum of 90 percent relative compaction, near the optimum moisture content. Fills thicker than 5 feet should be entirely compacted to at least 95 percent relative compaction. Due to the rocky nature of the on-site materials, it is expected that the majority of the excavated material will consist of over-sized (greater than 6 inches) rock which will require processing to break down prior to use in structural fills.

Fills should not consist primarily of rock, but rather the rock should be thoroughly mixed with on-site soils or approved import. This mixture should be moisture conditioned, placed in lifts not to exceed 8 inches thick, and compacted to a firm, dense state, comparable to 90 percent relative compaction as determined by the project geotechnical engineer. Fills containing appreciable amounts of rock will be difficult to test with conventional density testing methods. The contractor should anticipate preparing a test pad for determination of a performance specification for the placement rocky fill materials. Test pad preparation and determination of a performance specification are discussed below.

A representative of the geotechnical engineer should be present during all site clearing and grading operations to test and observe earthwork construction. This provision is especially important in cases where structural rock fills are to be constructed since compaction verification testing is not usually possible. The foregoing recommendations within this report are predicated upon our continued involvement in this project.

Test Pad, Acceptance Criteria

As stated above, the fills constructed with on-site materials will be rocky in nature and can be non-testable with conventional density testing methods. The contractor should prepare a test pad to determine the mix ratio of rock, rock-like material, and soil, as well as the suitability of proposed equipment. The Contractor should also develop a compaction qualification method which will achieve the minimum compaction criteria specified in this report.

The test pad should be large enough and deep enough to comparatively model the proposed fill section to be constructed. Where appropriate, representative samples of the fill should be acquired and sent to our laboratory to determine compaction properties and material gradation for quality assurance procedures in the field.

Slope Stability and Slope Construction

The greenstone and serpentine, which underlies the site are relatively high strength. Indications of slope instability, such as previous sliding or slumping on existing slopes, were not evident in the vicinity of the site. Due to the relatively dense nature of the materials underlying the site and the relatively low to moderate topographic relief within the site, the potential for landsliding or general slope instability is considered to be low unless unstable slopes are constructed during site improvement.

We recommend all cut and fill slopes be constructed with surface drainage and intermediate benches in accordance with the most recent edition of the Uniform Building Code. Deep-seated slope failures should not occur in cut and fill slopes that are designed and constructed in accordance with the recommendations provided in this report. Shallow slope failures such as surficial sloughing and flows, however, could still occur as a result of erosion and unanticipated water infiltration. To decrease the probability for shallow failures, the drainage and erosion control recommendations presented in this report should be implemented into the design and construction of the site. The implemented drainage and erosion control measures should be maintained during and after construction.

Fill slopes should be constructed at a gradient no steeper than 2:1 (horizontal to vertical). Fill slopes should be overbuilt and cut back to finish grade. Track-walking is not an acceptable method of fill slope construction or compaction, but can be used to firm and finish the final 4 to 6 inches of fill on the slope.

Slopes cut into the rock underlying the site may be constructed at a gradient of 1:1 or flatter. If necessary, it may be acceptable to construct steeper cut slopes but this should be reviewed by the project geotechnical engineer on a case-by-case basis.

Where fills are placed on slopes steeper than 6:1, the fills should be keyed a minimum of 5 feet into competent, undisturbed native soil or 3 feet into competent, undisturbed rock. Keyways should be a minimum of 10 feet wide and a subdrain should be placed at the bottom and to the rear of each keyway. The keyway should be sloped toward the back of the key at 2 percent or steeper. Benching of the subgrade should be performed above the toe of the fill. A bench and a subdrain should be provided for approximately every 10 vertical feet of elevation gain, and the bench should extend at least 1 foot into competent soil or rock. Where fill is to be placed within swales, subdrains should be installed at the base of the fill in a branch-like configuration along the centerline of the swale. Where fills are placed on slopes steeper than 6:1, the fills should be keyed a minimum of 5 feet into competent, undisturbed native soil or 3 feet into competent, undisturbed rock. Keyways should be a minimum of 10 feet wide and a subdrain should be placed at the bottom and to the rear of each keyway. The keyway should be sloped toward the back of the key at 2 percent or steeper. Benching of the subgrade should be performed above the toe of the fill. A bench and a subdrain should be provided for approximately every 10 vertical feet of elevation gain, and the bench should extend at least 1 foot into competent soil or rock. Where fill is to be placed within swales, subdrains should be installed at the base of the fill in a branch-like configuration along the centerline of the swale. Subdrains should be designed and constructed in accordance with the recommendations provided in *Subsurface Drainage*, below. The actual extent of the keying, benching, and subdrainage should be determined in the field at the time of construction by the Certified Engineering Geologist and the Geotechnical Engineer.

Surface water should not be allowed to flow over the top of slopes or down slope faces. Ponding of surface water should not be allowed at the top or bottoms of slopes, adjacent to retaining walls, foundations, or on pavement. Positive surface gradients of at least 2 percent should be provided adjacent to retaining walls and foundations to direct surface water toward suitable discharge facilities. Areas above slopes should be graded to a 2 percent gradient or greater to direct surface water away from the top of slopes toward a suitable point of discharge such as concrete lined ditches or surface drain inlets. Roof downspouts from buildings should be connected to solid pipes that transmit storm water onto paved roadways, into drainage inlets, or into storm drains. Collected water should not be allowed to flow onto slopes.

Landscaping drainage inlets should be provided around the proposed residences that adequately collect irrigation water and direct the water onto pavement or into storm water systems. It is imperative that the drainage inlets be properly designed and constructed so that the moisture content of the soils surrounding the slab-on-grade foundations do not become elevated and no ponding of water occurs. The design of the slab-on-grade foundations is based upon a well-drained condition. If the moisture content of the soils surrounding the slab-on-grade foundations, or the moisture content of the soils located below the slab-on-grade foundations, become elevated or excessively low, then mitigation measures will need to be implemented. Elevated or excessively low moisture contents of soils located near or below foundations may result in differential movement of the foundations.

All V-ditches should be appropriately sized for maximum storm water flows based upon the upslope tributary area and should discharge to appropriately sized drainage inlets. The concrete V-ditches should be adequately reinforced. Concrete V-ditches should be installed with the lip of the gutter cut at least 2-inches below adjacent surface grade. Forming and backfilling around V-ditches should not be allowed. Provisions should be made for the long-term maintenance of the site's surface drainage system, including removal of accumulated debris in V-ditches and sealing of cracks. Any damage to the drainage system should be repaired in an expedient manner to eliminate the possibility of concentrating surface flow and causing erosion. All V-ditches should be underlain by a subdrain. Construction of subdrains is presented below in *Subsurface Drainage.*

Subsurface Drainage

In order to minimize the potential for subsurface water induced problems, we recommend subsurface drainage be installed below or adjacent to proposed improvements including adjacent to the upslope edges of roadways, at the top and toe of slopes, below fill slopes, below interceptor ditches, within infilled swales or hollows, and below embankments. Specific subdraining requirement should be determined once the grading plans are prepared. Additional subdrainage will be necessary in areas of encountered or anticipated seepage. The actual location of the subdrainage should be determined in the field at the time of construction by the Certified Engineering Geologist and Geotechnical Engineer.

Wherever subdrains are located within fills, swales, or hollows, they should be spaced at no more than 10 feet vertically and 50 feet horizontally. Subdrains should consist of perforated pipe surrounded by free draining, uniformly graded, ½ to ¾ inch crushed gravel wrapped in filter fabric such as Mirafi 140N or equivalent. The filter fabric should overlap approximately 12 inches or more at joints. Alternatively, Caltrans Class 2 permeable material may be used in lieu of crushed gravel and filter fabric. Subdrain pipes should consist of rigid ABS (SDR-35) or PVC A-2000 (or equal) for fills less than 20 feet in height, ABS (SDR-23.5) or PVC Schedule 40 (or equal) for fills 20 to 50 feet in height, and ABS (SDR-15.3) or PVC Schedule 80 (or equal) for fills greater than 50 feet in height. The lateral drain pipes should be at least 4 inches in diameter. Laterals should be connected to a collector pipe with a minimum diameter of 6 inches. Subdrain clean-outs should be provided. The clean-out locations should be based upon the reach of the rotary cleaning systems and the restrictions of pipe bends. Subdrain trenches should be at least 18 inches wide and 4 feet deep. Wherever the subdrain extends to the ground surface and is not covered with a concrete V-ditch, then we recommend the subdrain be covered with a 12 inch thick clay cap compacted to at least 90 percent relative compaction. Onsite clayey soils can be used for the clay cap. Collector pipes should outlet to appropriate discharge facilities such as storm drains, drainage inlets, or storm drain manholes.

Setbacks

Cut and fill slopes should be setback from the site boundaries in accordance with the Uniform Building Code. Structures should be set-back from slopes in accordance with City of Auburn requirements and the Uniform Building Code.

Where roadways are located adjacent to cut, fill or native slopes, we recommend the roadway pavement section be setback at least 5 feet from the top or toe of the adjacent slope.

Trenches

Excavation of the exploratory test pits with a conventional rubber-tired backhoe was very difficult and often limited when slightly weathered or unweathered rock was encountered . The backhoe could not penetrate the very dense, relatively unweathered rock. Trenching in the very dense rock (encountered at depths ranging between 2 to 5 feet bgs) will require heavy trenching equipment and/or blasting.

Trenching in the very dense moderately weathered rock can often be accomplished with a large excavator (CAT 245 or greater in size), possibly requiring a pneumatic rock point. Due to the hard nature of the underlying rock, trenching can be excavated with almost vertically cut side slopes. However, a concern during trenching is that it will require a significant amount of effort to excavate. This may cause the sides of the trench to become loose due to cracking of the formation and may require the removal of material in the trench walls which were broken loose during excavation. It should be anticipated that over-widening of the trenches may occur during excavation and that the excavated materials will be large in size. The stability of excavated slopes could be affected by planes of weakness within the slope material or by the presence of seasonal seepage. This may also create fill material that doesn't meet the size requirements stated above. The project geotechnical engineer should be present during excavation and observe the trenches during construction to determine if factors that could affect slope stability are encountered, and to delineate contacts between weathered and unweathered materials where transitional slope gradients may be required.

Dewatering is not anticipated to be necessary for installation of utility lines assuming that construction takes place in the drier months of the year when the surface soil is not saturated and there is no surface water on the site. If water is found in the trench, it should be removed with a sump prior to any utility installation.

Utility trenches should be backfilled with approved on site materials or an approved import. Material excavated on-site will have to be crushed and/or screened to a maximum size of 4 inches prior to using it as trench backfill. Trench backfill materials should be approved by the geotechnical engineer prior to use. Crushed or screened native material should have no rock-to-rock contact. Due to the anticipated rocky nature of trench backfill on the site, bedding materials should be extended at least 12 inches above the pipe before general trench backfill proceeds to reduce the potential for pipe damage.

Trench backfill should be compacted to a minimum of 90 percent relative compaction using mechanical methods. The upper 3 feet of trench backfill in pavement areas should be compacted to at least 95 percent relative compaction Compaction should be based on ASTM Test Method D1557. We recommend a maximum uncompacted lift thicknesses of 1 foot unless field performance testing can demonstrate adequate compaction of thicker lifts. Jetting is not an acceptable means of compaction. Untestable, rocky fill should be compacted based on performance specifications determined to be adequate by the project geotechnical engineer at the time of backfilling.

Utility trenches should be excavated and shored according to accepted engineering practice following the Occupational Safety and Health Administration (OSHA) standards by a contractor experienced in such work. The responsibility for the safety of open trenches should be borne by the contractor. Traffic and vibration adjacent to trench walls should be minimized.

General Erosion Control

The erosion potential of the soil on or near the surface of the subject site is considered to be low to moderate. Erosion control measures should be implemented during and after construction to minimize soil erosion. This can be accomplished during construction using the following methods:

1. Site grading should be scheduled to avoid periods of heavy rains whenever possible.

2. Temporary slopes should be maintained at the flattest possible gradient.

3. During the rainy season, exposed soil on sloping ground should be covered as soon as possible. Covers could consist of grass and/or mulch (straw, wood chips, manmade fibers, etc.).

4. Water flow over areas disturbed by grading should be minimized. This can be accomplished by placing temporary earth berms at the top of sloped areas.

5. Dust should be controlled by sprinkling areas of exposed soil with water.

6. If appropriate, debris basins should be constructed to trap debris and silt prior to entering drainage channels. Hay bales, straw wattles, and other filtration devices can be used as silt traps along drainage channels and at drop inlets.

Following construction, exposed soil should be vegetated (planted with grasses or shrubs) or covered with a mulch or erosion control fabric to minimize soil erosion. Concentrated flows should be directed away from slopes and be piped or channeled into suitable drainage facilities.

Foundations

Although site plans were not available for the proposed residential development, based on the site topography it is likely that this property will require substantial cuts and fills during the initial grading process. All previous recommendations should be followed prior to foundation installation once grading is complete, the residential structures may be found an either footings, end-bearing piers, or a structural slab foundation system.

Foundations should be setback from the top and toe of slopes in accordance with the latest Uniform Building Code. At least 10 feet of cover should be provided between the outer face of foundations and unretained slope faces, as measured laterally between slope faces and the foundations. Where less than 10 feet of cover exists, deepening of the edge of foundations using drilled piers may be necessary in order to achieve 10 feet of cover for buildings located near slope tops. Where foundations are located adjacent to utility trenches, the foundation bearing surface should bear below an imaginary 1.5 horizontal to 1 vertical plane extending upward from the bottom edge of the adjacent utility trench. Alternatively, the foundation reinforcing could be increased to span the area defined above assuming no soil support is provided.

Footings / End-Bearing Piers

The footings/end-bearing piers should be designed for an allowable bearing capacity of 4,000 or 8,000 pounds per square foot (psf), respectively (for dead plus live loads). The allowable bearing capacity of both the spread footings and drilled piers may be increased by 33 percent for transient loading such as wind or seismic. The depth of footings should be at least 18 inches below lowest adjacent grade and the end-bearing piers should be drilled at least 2 feet below the engineered fill layer. Continuous strip and isolated footings should be embedded a minimum of 12 inches into the prepared subgrade (native soil, rock, or engineered fill).

The above recommended bearing capacities are provided based upon the assumptions that the project buildings are lightly loaded structures and not anticipated to have any large concentrated or column loading and the footings are going to be continuously bearing on intact very dense unweathered rock and/or engineered fills. If larger bearing capacities are required to resist larger isolated loading conditions, they can be addressed on a case-by-case basis by our office to provide the project architects/structural engineers with a more economical footing design.

Minimum footing excavation depths may be reduced in accordance with the following recommendations when the very dense unweathered rock is encountered. Our representative should verify the bearing soils in footing and the drilling and clean out of piers in the field at the time of construction.

Where foundation excavations will require penetration of more than 6 inches into very dense rock, foundations may be excavated to a total depth of 6 inches and doweled into the underlying very dense rock. This condition is expected to occur where cuts for the building pads exceed 1 to 2 feet in depth and if over-excavation is not performed. The presence of the very dense slightly weathered or unweathered rock should be verified by the project geotechnical engineer prior to implementation of doweling.

Reinforcement of the footings should be determined by the design engineer. Footing excavations should be cleared of all loose soil and construction debris prior to the placement of concrete. Any soft areas within the footing bottoms should be removed until dense, unyielding conditions are encountered. The project geotechnical engineer should be allowed to observe footing excavations prior to placement of concrete or reinforcement in order to confirm anticipated soil conditions.

If foundations are designed in accordance with the recommendations above, we estimate total settlement for building foundations to be less than ½ inch. Differential settlements should be less than ¼ inch over a distance of approximately 30 to 50 feet.

Structural Slabs

As described previously, wherever granular soils, granular soil/rock mixtures, or bedrock will exist below the foundations to a depth of at least 5 feet, the residences can alternatively be founded on structural slab foundations.

Structural slab foundations should be designed in accordance with the parameters presented in the latest edition of the Uniform Building Code. The subgrade materials beneath the slabs should be considered to have an unconfined compressive strength of 1,000 pounds per square foot, and a Weighted Plasticity Index of 10 percent. Slabs should be at least 6 inches thick. In addition, the slabs should be designed to cantilever a minimum distance of 2 feet and free span a minimum of 5 feet.

Migration of moisture through the slab foundations should be minimized by providing a moisture barrier between the subgrade soils and the bottom of the slabs. We recommend the moisture barrier consist of 2 inches of lightly moistened sand overlying an impermeable membrane that is at least 10 mil thick. As an alternative to using a membrane, "moistop" may be used. Wherever a mat slab will be used, we recommend the moisture barrier be underlain by 4 inches of uniformly graded, free draining gravel. Free draining gravel is not necessary below post-tensioned slabs-on-grade.

A concrete barrier at least 12 inches wide or "thickened edge" that is supported directly on the subgrade materials should be provided at the perimeter of the slab foundations to provide a water cutoff for the moisture barrier. The edge should extend at least 2 inches below the moisture barrier. In addition, interior areas of the slab which support point or line loads should also be thickened a minimum of 12 inches and supported directly on the subgrade.

Concrete slabs retain moisture and often take many months to dry. We recommend that carpets that allow air to pass through them be used over concrete floor slabs. Additionally, if vinyl floor tiles or wood flooring are used, the concrete floor slab should be given sufficient time to air dry before the tiles or wood are applied. Alternatively, a floor sealant could be applied over the concrete to minimize moisture from accumulating under the flooring.

We recommend our firm review the foundation drawings and specifications prior to submittal to verify that the recommendations provided in this report have been used in the design of the slabs.

Lateral Resistance

The coefficient of friction to resist sliding is 0.40 and the minimum depth of footings should be 18 inches. Resistance to lateral loads may be provided by assuming a passive pressure based on an equivalent fluid weight of 400 pcf. In designing structures to resist lateral loads, the upper 12 inches of soil should be ignored and the lateral resistance of the soil should be limited to 2,500 psf.

Footing dowels could also be designed to resist anticipated lateral loads when footing are bearing into rock. A compressive strength of 200 psi can be assumed for very dense rock. As a minimum, doweling should consist of No. 6 rebar penetrating at least 36 inches into the very dense conglomerate. The holes for the dowels should be spaced no further apart than 3 feet on-center and should be at least 1.5 inches in diameter or twice the diameter of the dowel bar, whichever is larger. Dowels should be grouted into the very dense conglomerate using a high strength grout with a minimum compressive strength of 6,000 psi, or an approved epoxy. Isolated footings should incorporate a minimum of at least two dowels.

UBC Seismic Design Parameters

General seismic parameters for design of buildings in accordance with the 2001 California Building Code (CBC) are as follows:

- **The Seismic Zone Factor, Z:** 0.30 (CBC, 2001, Table 16-I).
- **Soil Profile Type: Type S_b** (CBC, 2001, Table 16-J).
- **Seismic Source Type: B** (CBC, 2001, Table 16 A-U).
- **Near Source Factor, N_a:** 1.3 (CBC, 2001, Table 16 A-S).
- **Near Source Factor, N_v:** 1.6 (CBC, 2001, Table 16 A-T).
- **Seismic Coefficient, C_a:** 0.33 (CBC, 2001, Table 16-Q).
- **Seismic Coefficient, C_v:** 0.45 (CBC, 2001, Table 16-R).

A peak horizontal ground acceleration (PHGA) of 0.14, based on the Design Basis Earthquake (DBE), should be used in the design where CBC parameters are not applicable and DBE design levels are required.

Exterior Slabs

Exterior slabs may be placed on properly prepared native subgrade or engineered fill materials, prepared in accordance with the grading recommendations previously presented. The subgrade for exterior flatwork should be free of any debris, uniformly compacted and thoroughly wetted before the concrete is placed. Reinforcement, as determined by the structural engineer, may be needed in areas subjected to unusually heavy loads.

Retaining Walls

The equivalent fluid weights provided in Table 1 may be used for design of drained retaining structures with horizontal backfill, respectfully. In cases where there will be sloping backfill, the values in Table 1 may be increased by 15 pcf. The drained condition assumes that the backfill behind the wall is adequately drained to avoid saturation and introduction of hydrostatic pressure. The values given should be considered as ultimate values since they do not include a factor of safety.

TABLE 1

EQUIVALENT FLUID WEIGHTS

Condition	Horizontal Backfill
Active Condition	35 pcf
Passive Condition	400 pcf
At-Rest Condition	60 pcf

In the design of retaining structures, if any surface loads are closer to the edge of the retaining wall than half of the height, then the design wall pressure should be increased by 0.30q over the whole area of the retaining wall. In this expression, q is the surface surcharge load in psf.

Positive drainage is essential behind any retaining wall to prevent the backfill from becoming completely saturated and the buildup of hydrostatic pressure. Positive drainage for retaining walls should consist of a vertical layer of permeable material, such as coarse sand or pea gravel at least 6 inches thick, positioned between the retaining wall and the backfill. If pea gravel is used, a non-woven filter fabric should be placed between it and the backfill to prevent the pea gravel from becoming clogged. A synthetic drainage fabric, such as Enkadrain or equivalent, may be substituted for the gravel or sandy layer, if desired. Care must be taken during installation to assure that the filter part of the material faces the backfill. Collected water may be removed either by installing weep holes along the bottom of the wall or by installing a perforated drainage pipe along the bottom of the permeable material continuously sloped towards suitable drainage facilities.

Retaining walls should be founded on footings or end bearing piers, in accordance with the recommendations presented above.

Flag and Light Poles

Foundations for flagpoles and other pole-supported structures may be designed using the formula in the California Building Code (2001, Section 1806A). An allowable lateral soil-bearing pressure of 300 psf per foot of depth is applicable to the site soil. Where flag or light poles will not be adversely affected by ½-inch of lateral motion at the ground surface due to short-term lateral loading, an allowable lateral soil-bearing pressure of 600 psf per foot of depth is applicable.

Foundation depths for pole supported structures could be reduced and lateral bearing pressures can be increased when foundations are constructed in undisturbed very dense rock underlying the site. Deeper than 3 feet, an allowable lateral bearing pressure of 500 psf per foot of depth is applicable for pole-supported structures founded in the undisturbed volcanic conglomerate and an allowable lateral bearing pressure of 1,000 psf per foot of depth is applicable where the structure will not be adversely affected by ½-inch of lateral motion at the ground surface due to short-term lateral loading. The presence of the conglomerate should be verified by the project geotechnical engineer and fill at the location should not exceed 1 foot unless fill is accounted for in the design of the foundation.

It will be difficult to construct the drilled shafts in the unweathered rock due to the very hard nature of the materials. As an alternate foundation system, spread footings with dowels may be used to support the proposed pole structures. Refer to the "Foundations" and "Lateral Earth Pressures" sections above for bearing capacity and formation design details.

Pavement

One sample of the surface soils were combined and tested for R-value. These materials consisted of clayey sands with weathered rock, and had R-value of 42. However, due to the variability of the soil, an R-value of 30 was used for design. Once the site grades have been established, EGC should review the Pavement Plan/Specifications to assure compliance with our recommendations.

Flexible Pavement

We developed the following alternative preliminary pavement sections using Topic 608 of the State of California Department of Transportation Highway Design Manual, an R-value of 30, and assumed traffic indices. Recommended pavement structural sections are included in Table 2. The TI is a measure of wheel load, frequency, and intensity. It has been our experience that a TI of 4.5 is often used for cul-de-sacs and automobile parking, 5.0 for automobile traffic, 5.5 for limited truck traffic, and of 6.5 for channelized flow and bus/truck traffic. Use of the proper TI should be confirmed by the project civil engineer. It should be noted that if construction traffic will drive on the finished pavement sections, the design TI may not be adequate for a full service life. Adjustments to the TI may be necessary to accommodate construction traffic. If imported soils are used to raise site grades, confirming R-value tests should be performed on imported soils planned for pavement surfaces.

TABLE 2

RECOMMENDED PAVEMENT SECTIONS		
T.I.	**Asphalt Concrete (inches)**	**Class 2 Aggregate Base (inches)**
4.5	2.5	6.0*
5.0	2.5	7.0
5.5	3.0	7.0
6.5	3.5	9.0

* minimum thickness value

Rigid Pavement

Rigid pavements, generally used for vehicular buses and truck traffic, should consist of Portland Cement Concrete (PCC) underlain by a minimum of 6 inches of Caltrans Class 2 aggregate base. For TIs up to 5.0, the concrete should be at least 5 inches thick. For TIs from 5.0 to 6.5, the concrete should be at least

6 inches thick. For the PCC sections, a minimum reinforcement of No. 4 bars on 18 inch centers should be considered. If large loads from trucks/buses turning or excessive bus traffic is expected (TI=7.0 or greater), then we should be consulted to provide further rigid pavement recommendations and the project structural engineer should design the slab and detail the rebar for the anticipated wheel loadings.

General

These pavement sections are based on the assumption that the native subgrade soil or fill is uniformly compacted to at least 95 percent relative compaction and that the baserock is uniformly compacted to at least 95 percent relative compaction. Pavement areas should be sloped to allow for positive surface drainage. Adequate surface slope, subgrade crown, and uniform compaction contribute to long-term pavement performance.

The drainage of pavement areas should be designed so that water is not allowed under the paved areas. If water is trapped under paving the water can saturate the base course and soil subgrade which could result in premature pavement failures.

Cutoff curbs should be installed where pavement abuts landscaped areas. These cutoff curbs should extend to a minimum depth of 2 inches below the section subgrade or to in-place rock, whichever depth is less, to reduce the amount of water that can seep beneath the pavement. Where cutoff curbs are undesirable, subgrade drains can be constructed to remove excess water from landscape areas or an impermeable barrier, such as 20 mil HDPE, could be placed at the back of curb to a depth of approximately 1 foot below subgrade.

Where drop inlets, storm drain manholes, or other surface drainage devices are to be installed, we recommend that slots or weep holes be provided at the bottom of the base course (immediately above the soil or rock subgrade) to allow for drainage of the water which might become trapped in the structural baserock section. The slots or weep holes should be screened with a fine wire mesh or geotextile to prevent the migration of fine material out of the baserock.

LIMITATIONS

The analyses, conclusions, and recommendations contained in our report are based on site conditions as they existed at the time of our study, and further assume that probes such as exploratory borings are representative of the subsurface conditions throughout the site; i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the probes.

Soil conditions cannot be fully determined by test pits and seismic refraction surveys commonly encountered. Such unexpected soil conditions often require that additional expenditures be made to attain a properly constructed project. Some contingency fund is therefore recommended to accommodate such potential extra costs.

A determination of flooding potential or the existence of wetlands was beyond the scope of this report.

This geotechnical investigation study did not include an investigation regarding the existence, location, or type of possible hazardous materials. If any hazardous materials are encountered during construction of the project, the proper regulatory officials should be notified immediately.

REFERENCES

California Building Code, 2001, Section 1806A

Hart, E.W., 1997 (Revised 2003), Fault Rupture Hazard Zones in California: California Division of Mines and Geology Special Publication 42.

Helley, D.J. and Harwood, D.S., 1985, Geologic Map of the Late Cenozoic Deposits of the Sacramento Valley and Northern Sierran Foothills. California: U.S. Geological Survey, Map MF-1790.

Jennings, C.W., 1994, Fault Activity Map of California and Adjacent Areas with location and ages of Recent Volcanic Eruption.

Livingston, J.G., 1974, Engineering Geologic Map of the Auburn Quad, Placer County, California; Placer County Planning Department.

Wagner, D.L., et al, 1981, Geologic Map of the Sacramento Quadrangle; DMG Regional Geologic Map Series, Map No. 1A.

March 29, 2005

Ms. Leah Rosasco
Placer County Planning Department
11414 B Avenue
Auburn, CA 95603

Subject: **PEER REVIEW OF ENVIRONMENTAL DOCUMENTS**
Proposed Fiddler Green Subdivision
Auburn, California

References:

Twining Laboratories, Inc., “Phase I Environmental Site Assessment, Highway 49 and Luther Road, Auburn, California”, for Psomas and Associates, December 15, 1993.

Charles Lockwood Consulting Engineering, Inc., “Phase I Environmental Site Assessment, Bohemia Parcels”, June 8, 2004.

España Geotechnical Consulting, “Draft Geotechnical Investigation Report for the Proposed Bohemia Residential Development, Auburn, California”, for Tim Lewis Communities, November 24, 2003.

España Geotechnical Consulting, “Phase I Environmental Site Assessment Update for the Proposed Bohemia Residential Development, Auburn, California”, for Tim Lewis Communities, November 2004.

Vector Engineering, Inc., “Workplan for Remediation of Bohemia’s Former Auburn Planer Mill Site”, for Bohemia, Inc., July 1991.

Vector Engineering, Inc., “Final Status Report on Former Planer Mill Site, Highway 49 at Luther Road, Auburn, California”, for Bohemia, Inc., December 9, 1991.

California Regional Water Quality Control Board – Central Valley Region, “Concurrence with Placer County Request for Site Closure, Bohemia Site, Auburn”, to Bohemia, Inc., October 29, 1992.

Dear Ms. Rosasco:

P&D Consultants (P&D) has completed a peer review of environmental documents as listed above, pertaining to the proposed Fiddler Green Subdivision development. Previous environmental work on the site, which comprises approximately 18.54 acres in an area east of Highway 49 and north of Luther Road, north of the City of Auburn, has indicated a potential environmental concern in that the Property was formerly occupied by a lumber mill operated by Bohemia, Inc. The available environmental documentation generally falls into two categories: Phase I Environmental Site Assessments or Updates on the subject site itself, and remediation reports on another area, southwest of the subject Property but also part of the former Bohemia Inc. facility, where diesel-impacted soil, shallow bedrock and groundwater had been identified and remediated.

The review of the documents indicated the following:

Environmental Site Assessments on Subject Site

Twining Laboratories, 1993

At the time of the 1993 Phase I Environmental Site Assessment, the Property had been vacant land since 1985, when the Bohemia lumber mill closed and all on-site buildings were demolished. The portion of the Bohemia site comprising the subject Property had been used for dry kilns, saw shops, and inside and outside storage. The Property was found to be an irregularly-shaped parcel of land bordered on the east by Canal Street, on the north by a residential area and a set of railroad tracks, on the west and southwest by Wise Canal, and on the south by vacant land and a Pacific Gas & Electric storage yard. Fiddlers Green Canal traverses the center of the Property in a generally north-south direction, turning east near the property line with Pacific Gas and Electric.

The 1993 Phase I report was prepared before promulgation of the first ASTM standard for Phase I assessments (ASTM E1527-94), and therefore does not reference the standard. The report includes review of aerial photographs dated 1952, 1958, 1962, 1967, 1988 and 1992. No city directory or Sanborn map coverage was found, nor would such coverage be expected in this area.

Indications of the presence of imported fill material (mostly wood debris and sawdust) were noted, based on the topography of the site and the results of previous geotechnical investigations (drilling) on the Property. Several soil piles, 55-gallon drums with unknown contents, and cut-off pipes of uncertain origin were noted on the Property.

The Twining report included the following conclusions:

- The potential for impact to soil or groundwater from surface debris observed on site appeared to be low.
- The potential for impact to soil or groundwater from imported fill material was unknown.
- The potential for impact to soil or groundwater from the pipes and soil piles was unknown.
- The potential for impact to soil or groundwater from offsite sources appeared to be low.

The Twining report included the following recommendations:

- Additional assessment by excavation of the fill material, soil piles, and pipes.
- Removal of the 55-gallon drums.

The recommendations of the 1993 Twining report do not appear to have been carried out, based on the recurrence of the same concerns and recommendations in later reports.

Charles Lockwood, Consulting Engineer, June 2004

This report was prepared for Western Sierra Bank. At the time of report preparation the site was still vacant land, as it had been since 1985. During preparation of the report, the consultant reviewed aerial photographs from 1962, 1971, 1982, 1989 and 1993. The buildings were no longer visible after the 1982 photograph. Former site usage was found to be as for wood staging and sawmill operations. No underground storage tanks or indications of the former presence of such tanks were found, although the leaking underground storage tank case on the portion of the old Bohemia facility west of the subject site was noted.

Items of environmental concern noted during the inspection included the following:

- Two pits with protruding pipes, possibly asbestos-wrapped, in the northern part of the site.
- Approximately 50 old tires at various locations on the Property.

- Eight old drums (contents unknown) on the south side of the Property.

The Lockwood report concluded that based on limited historical information it appeared that the Bohemia facility had operated on site from approximately 1901 to circa 1982/83. No underground storage tanks were identified. Old tires, drums and wood debris were noted.

The Lockwood report did not have a Recommendations section, and thus did not recommend further action. The report did state, “In our opinion, historic use as a lumber mill represents a recognized environmental condition in connection with the property.”

<u>España Geotechnical Consulting, Phase I Update, November 2004</u>

This report references and updates the Phase I Environmental Site Assessment performed on the site by Charles Lockwood, Consulting Engineer in June 2004. The España report notes that it concurs with the findings and conclusions of the Lockwood report. The España report was prepared in support of proposed residential development of the site by Tim Lewis Communities.

At the time of the España report, the subject Property remained vacant land. No changes to land use appear to have occurred since the earlier assessments in 1993 and 2004. The España report again notes the presence of soil piles, pipes of unknown purpose, and 55-gallon drums with unknown contents on the site.

The cover letter to the report states in part, “…historic use of the property as a lumber mill represents a recognized environmental condition.” However, the Conclusions section states, “No known or potentially hazardous materials issues that are likely to present a “Fatal Flaw’ impact to the project were identified as occurring either within or immediately adjacent to the study area.”

The recommendations of the Update report were as follows:

- A Limited Phase II Environmental Site Assessment consisting of sampling of surface and near-surface soils should be performed in the area of the former lumber mill buildings, to determine potential impacts from wood preservatives or petroleum hydrocarbons.
- The contents of the 55-gallon drums should be determined and the drums disposed of appropriately.
- Naturally-occurring asbestos (NOA) was likely present in the serpentine bedrock underlying the site. Recommendations for this material were to be made in a concurrent geotechnical report. (Laboratory testing of rock samples in the geotechnical report did not indicate concentrations of asbestos above trace levels, and the geotechnical report concluded that special grading requirements would likely not be needed for the site).

<u>Review</u>

The previous Phase I reports indicate a potential environmental concern regarding past site usage as a lumber mill, and the presence of imported fill material, soil piles of unknown content, drums of unknown content, and pipes of unknown purpose. In the Lockwood report and the cover letter to the España report, the former site usage as a lumber mill is referred to as a “recognized environmental condition”. ASTM Practice E1527-00 states in part:

> “The term recognized environmental condition means the presence or likely presence of any hazardous substances or petroleum products on a property under conditions that indicate an existing release, a past release, or a material threat of a release of any hazardous substances or petroleum products into structures on the property or into the ground, groundwater, or surface water of the property. The term includes hazardous substances or petroleum products even under conditions in compliance with laws.”

Given that indications of a release of hazardous materials at this site have not been documented, we question whether the former site usage alone inherently constitutes a Recognized Environmental Condition (REC). The term Recognized Environmental Condition is generally applied to situations where a release or probable release has been documented.

However, in view of the former site usage and the presence of several suspect items, apparently remaining from former operations on the Property, we concur with the recommendations of the 1993 and 2004 (España) reports that a Phase II Environmental Site Assessment should be performed in the suspect areas, including the former site buildings, soil piles and drums. Also, the drums, which have apparently been on site for at least twelve years and perhaps twenty years or more, should be sampled as to their contents and disposed of appropriately.

Based on the lack of changes in site conditions between the 1993 and 2004 reports and the recent date of the 2004 reports, no additional Phase I assessment of this site appears to be warranted at this time.

Remedial Activities on Adjacent Site

Neither the 1993 nor the 2004 Phase I report contains more than a passing reference to the 1991-1992 remediation of diesel-impacted soil, shallow bedrock and groundwater at an adjacent part of the Bohemia site, southwest of the subject Property across the Wise Canal. The diesel contamination was found during the removal of three underground storage tanks (two 10,000-gallon diesel USTs and one 2,100 waste-oil UST) in 1986 and 1988. The remediation was performed under the direction of the California Regional Water Quality Control Board, Central Valley Region, and the Placer County Department of Environmental Health.

Releases of diesel from the USTs were found to have impacted the underlying soil and to have penetrated to shallow bedrock (consisting of fractured ultramafic rock) and groundwater, which occurs in the bedrock. Eleven groundwater monitoring wells were installed in the early 1990s in the area of the removed USTs. In 1991, Vector Engineering excavated approximately 2,500 cubic yards of diesel-impacted soil and bedrock from two areas around the former tanks. Some diesel-impacted bedrock remained in place since the rock was too hard to be removed. Three groundwater monitoring wells remained on the site and were sampled on a quarterly basis thereafter. After five quarters of groundwater sampling, the case was closed with the concurrence of the Regional Water Quality Control Board in October 1992, and the remaining wells abandoned thereafter.

Review

This case is mentioned in the 2004 reports. Although some diesel-impacted bedrock was left in place, given the current status of the case, the downgradient location, and the lack of detectable groundwater impact in pre-closure sampling, the potential for impact to the subject Property from this adjacent site appears to be very low.

SUMMARY

The previous Phase I Environmental Site Assessments carried out on the subject site, considered as Phase I assessments, appear to be adequate to characterize historical site usage, present site conditions and potential environmental concerns to the extent they can be determined without invasive sampling. However, unless it can be shown that the recommendations of the previous Phase I reports have been carried out, the Phase II Environmental Site Assessment described in the 2004 Update report should be performed in order to characterize any environmental constraints or concerns related to historical site usage.

If you have any questions regarding this review, please contact the undersigned at (714) 648-2074.

Sincerely,

Robert R. Olsen, R.G., R.E.A. II
Senior Geologist

cc: P. Choi

ENGEO INCORPORATED

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS
MATERIALS TESTING

Project No.
6744.5.001.01

July 15, 2005

Mr. Randy Chafin
P&D Consultants
3840 Rosin Court, Suite 130
Sacramento, CA 95834-1699

Subject: Fiddler Green EIR
Auburn, California

GEOTECHNICAL REPORT REVIEW

References:

1. Espana Geotechnical Consulting; Geotechnical Investigation Report, Bohemia Residential Development, Auburn, California; December 2004; Project No. P201.
2. Wallace Kuhl & Associates; Geotechnical Engineering Report, Wal-Mart Auburn Store, Highway 49 near Luther Road, Auburn, California; September 16, 1992; WKA No. 2201.01.

Dear Mr. Chafin:

At your request, we reviewed the referenced geotechnical reports prepared for the subject property along State Route 49 in Auburn, California. The purpose of our review was to provide our professional opinion regarding the adequacy of the work contained within Reference 1 for the proposed development. Reference 2 has been reviewed primarily to gain additional insight into the subsurface soil and rock conditions present onsite.

In addition to reviewing the above referenced reports, we have reviewed the Placer County Grading Ordinance and have visited the site.

General Site Description

The 18-acre site is located off of Canal Road, near Luther Road near the City of Auburn, California. The property generally slopes downhill to the west towards State Route 49 and contains a series of terraces. The eastern half of the property is covered with weeds and grasses and occasional trees. The western portion of the property contains weathered asphalt pavements and old slabs and foundations, likely from previous lumber mill operations onsite. Fiddler Green Canal crosses the central portion of the property in a north-south direction. Wise Canal parallels the easterly property line.

Published topographic and geologic maps show the properties to be situated at an elevation of roughly 1450 feet above mean sea level (msl) and underlain by Paleozoic to Mesozoic metavolcanic rocks (greenstone).

Proposed Development

We understand the site will be developed into 110 single family homes. We anticipate the buildings will be one- and two-story wood-framed homes; therefore, building loads are expected to be relatively light to moderate. Roadways and underground utilities are also planned to service the new development.

Relatively substantial cuts and fills may be necessary to grade the site, along with the relocation of Fiddler Green Canal. Some retaining walls may also be required.

Prior Geotechnical Study

Espana Geotechnical Consulting (EGC) conducted a geotechnical exploration at the site in December 2004. The exploration included the excavation of 16 test pits, a seismic refraction survey, and limited laboratory testing.

The test pits performed for the project geotechnical report generally encountered ½ to 5 feet of red clayey sand/sandy clay with rock fragments. Underlying the near surface soil deposits, the test pits generally encountered completely weathered/sheared to unweathered greenstone.

Undocumented fill materials were encountered near the south central portion of the property. The exploration logs indicate that the fill is on the order of 3 to 4 feet thick at the locations explored. The fill includes loose, clayey and silty sands with organic debris and concrete rubble.

Conclusions

In general, it is our opinion that the referenced EGC report addresses the basic geotechnical considerations required for development. We summarize below our specific comments regarding the EGC report that may require additional analysis, discussion, and/or modification to project recommendations/specifications, as appropriate.

- Following grading, several lots may be founded entirely within cut. The rock indicated to be present onsite is very hard, and as a result, excavations for plumbing and footings will be very difficult on these pads. Furthermore, because the cut areas are likely relatively impermeable, water can pond just below your slabs-on-grade, potentially leading to slab moisture problems. We recommend that cut pads within rock be over excavated to a minimum depth of 1½ feet to facilitate drainage and excavation. This is a common geotechnical recommendation where cuts for building pads expose hard materials.

- The ECG report provides recommendations for both footings/end bearing pier foundations and structural mat slabs. We did not find any specific recommendations regarding non-structural interior slabs-on-grade. It may also be beneficial to develop design criteria for post-tensioned mat foundations.

  The geotechnical report outlines the preparation of a test pad for development of a performance specification for non testable fill material. We suggest that preliminary performance specification criterion be developed for non testable fill. This would likely include a minimum number of passes, lift thickness, moisture requirements, and minimum equipment size to obtain adequate compaction. The contractors test pad and proposed compaction methods should be reviewed and approved by the project geotechnical engineer prior to implementation.

As noted in the listed items above, minor modifications to some of the report recommendations may be appropriate once final construction plans are prepared and reviewed by ENGEO.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied.

If you have any questions regarding this letter, please call and we would be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Jonathan C. Boland, PE

Reviewed by:

REGISTERED PROFESSIONAL ENGINEER
MARK M. GILBERT
NO. 2191
Exp. 3-31-06
GEOTECHNICAL
STATE OF CALIFORNIA

for Mark M. Gilbert, GE